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SEISMIC PERFORMANCE OF EXISTING NEW ZEALAND SHEAR WALL STRUCTURES

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Abstract

Assessment of the structural performance of existing buildings requires a better understanding of seismic performance of the structural components designed according to different versions of design codes. This study provides a summary of the evolution of the reinforced concrete wall design provisions in New Zealand, and investigates their effect on seismic performance of structural walls. For this purpose, a typical rectangular wall is designed according to different versions of New Zealand concrete design standards, and a finite element approach is used for numerical simulation of the walls subject to cyclic loading. The modeling approach has been verified using experimental results of walls with different shear-span ratios which failed in different modes. Performance of the designed wall models is investigated in terms of failure pattern, drift capacity and displacement as well as curvature ductility. Seismic performance of the walls designed according to the previous versions of NZ design codes will provide a considerable contribution to better understanding of the wall capacity in seismic assessment of existing buildings.

Introduction

Structural walls (also referred as shear walls) are one of the common lateral load resisting elements in reinforced concrete (RC) buildings in seismic regions. According to the Canterbury Earthquakes Royal Commission Reports (2012), structural walls in Christchurch buildings did not perform as anticipated in the 2010-11 series of Canterbury earthquakes. Boundary zone crushing and bar buckling were observed in pre-1970s RC walls which were generally lightly reinforced, were not detailed for ductility and had inadequate reinforcement to provide confinement to the core concrete and buckling restraint to the longitudinal reinforcement. On the other hand, modern (post-1970s) RC wall buildings were observed to have experienced failure patterns like wall web buckling, boundary zone bar fracture and buckling failure of ducted splice. In a number of cases, compression failure occurred in the outstanding legs of T and L walls in addition to out-of-plane displacements, thereby resulting in overall buckling of the wall. In some cases, transverse reinforcement spacing did not meet the code requirement to prevent buckling of the longitudinal (vertical) reinforcement, and bar buckling resulted in high localized strains and decreased the tensile strain capacity.

Figure 1 shows some examples of different failure modes, observed in RC walls in the 2011 Christchurch earthquake. As a result of the unexpected performance of shear walls in the 2010-11 Canterbury earthquakes, some issues have been identified to be further investigated (CERC 2012). The main issues lie around the buckling of bars, out-of plane deformation of the wall (especially the zone deteriorated in compression), and reinforcement found snapped beneath a single thin (in terms of residual value) crack.

The performance of RC structural walls in recent earthquakes has exposed some problems with the existing design of RC structural walls, leading to a call for the revision and improvement of current wall design procedures (Sritharan et al. 2014). In order to gain a better understanding of the seismic behaviour of shear walls underpinning the preparation of amendments to current code provisions, it is necessary to investigate the seismic response of walls including the causes of different failure modes observed in the recent earthquakes. As repeated experimental investigation is too demanding, a more plausible way to scrutinize the observed performance of RC shear walls against their expected performance is to simulate the walls using an efficient numerical model. This study investigates the ability and robustness of a finite element model in predicting nonlinear behavior and failure patterns of walls. From published literature, experimental results of walls with different shear-span ratios which failed in different modes are used for the verification of the model.

In this study, evolution of the reinforced concrete wall design provisions in New Zealand concrete structures standard is studied, and seismic performance of a wall designed according to these provisions is investigated.

(a) (b) (c)

Figure 1. Wall damage in the the 2010-11 Canterbury earthquake sequence: (a) buckling of well-confined wall (Elwood 2013); (b) bar buckling and fracture (Kam et al. 2011); (c) shear-axial failure (Kam et al. 2011)

History of New Zealand wall design provisions

A summary of the of the reinforced concrete wall design provisions required by different versions of the New Zealand concrete structures standard (NZSS1900 1964, NZS3101 1970, NZS3101 1982, NZS3101 1995, NZS3101 2006) is given in the Appendix, Table 4.

Typical wall design

A typical rectangular wall is designed according to the wall design provisions given by different versions of New Zealand concrete structures standard to study their effect on seismic performance of structural walls. The wall is designed in accordance with provisions of NZS3101:2006, and redesigned according to the provisions specified in its former versions such that the flexural capacities provided by the wall sections are alike. There are no specific seismic provisions in the 1964 version of the concrete code, and is not included in the design. General properties of the designed walls and the design demands are given in Table 1 and Table 2, respectively. Table 3 displays section geometry and reinforcement configuration of the designed walls.

Table 1. General properties of the designed walls

Story Height (mm)	No. of Stories	Hw (mm)	Heff (mm)	Lw (mm)	fc (Mpa)	fy (Mpa)
4000	4	16000	12000	4000	35	300

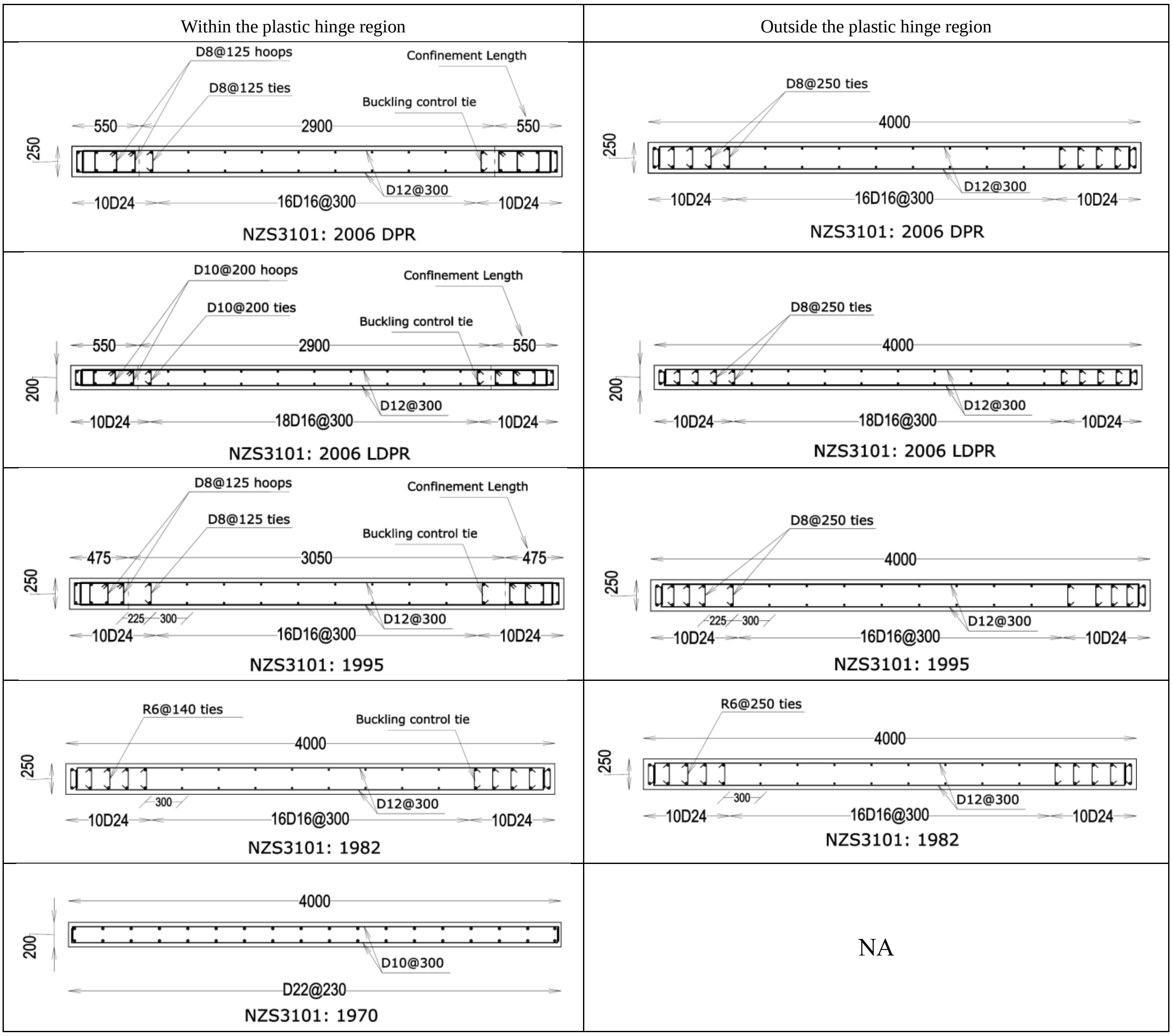
Table 2. Design demands

Shear (kN)	Moment (kN.m)	Axial (kN)
792	9500	1750

Considering the fact that the walls have been designed to provide fairly close flexural capacities, longitudinal reinforcement configuration does not have a big variation among the designed walls. Therefore, the parameters changing in the designed walls are wall thickness and detailing of the boundary region, i.e. confinement and anti-buckling reinforcement requirements as well as shear reinforcement which happened to be identical in the walls designed based on the capacity design concept (from 1982 onward)

confinement eligibility.

Table 3. Wall sections



Wall thickness has been chosen based on the minimum requirements of the codes except for the 1982 design which is described in the following section. As can be observed in Table 4, in addition to the general minimum thickness, the wall thickness is limited by other limitations, namely limitations on the height to thickness ratio and minimum thickness for prevention of instability within plastic hinge region. The limiting thickness for the 2006 and 1995 designs is the one for prevention of wall instability which was not required in older versions of NZS3101 (i.e., 1982 and 1970).

The height to thickness ratio requirement of the 1982 code ($\frac{L_n}{t} \leq 10$) would result in a considerably bigger wall thickness. However, as mentioned in Table 4, this provision need not be satisfied if the neutral axis depth for the design loading is less than $0.3l_w$. As a result, this limitation does not apply and the thickness has been chosen according to the general minimum thickness requirement. The minimum wall thickness required by the 1970 code is 6in. (152.4mm), and 200mm has been adopted.

The provisions for confinement of the boundary zone in plastic hinge region of walls have been required since the 1982 version of the concrete structures standard. These requirements apply when the computed neutral axis depth of the section in the potential yield regions exceeds a threshold, namely c_c (Table 4). It is considered that in such a situation the normally assumed concrete compression strain at the extreme fibre of a section may not be sufficient to ensure adequate ductility of the section (NZS3101 1982). For the wall investigated in this study, the provisions of the 1984 code result in a relatively (about two times) bigger value of c_c when compared to the later versions of the concrete code (1995 and 2006). The 2006 code provisions give a slightly smaller value of c_c than the ones of the 1995 code. Therefore, the neutral axis depth at the nominal flexural strength limit state is less than the value of c_c for the 1982 design resulting in exemption of the wall boundary region from confinement reinforcement, and only anti-buckling reinforcement requirements apply.

Figure 2 displays different design alternatives complying with the 1982 code. As can be seen in this figure, if the minimum thickness required by the code is adopted for the 1982 design (200mm, Option 1), the confinement requirements apply as the neutral axis position shifts beyond the value of c_c. Also, for the thickness of 250mm, uniformly distributed longitudinal reinforcement (Option 3) would result in a deeper neutral axis position compared to the wall that has higher reinforcement ratio positioned in the boundary regions (Option 2), and therefore requiring confinement provisions to be satisfied. However, if a thickness of 400mm is adopted (Option 4) the neutral axis position would not be deep enough to meet the confinement eligibility.

As can be seen in Figure 2, In cases where the confinement reinforcement is required (Options 1&3), the total effective area of hoop bars and supplementary cross ties within spacing s_h is much greater than the one calculated for the later versions of the concrete design standard (Table 3).

The amount of transverse reinforcement in the boundary region shall be calculated using Equation 1 and Equation 2 for the 1982 and 2006 concrete codes, respectively. The amount of A_{sh} calculated using Equation 1 is much greater than the one of the 2006 code (Equation 2) considering the term $\left(0.5 + 0.9\frac{c}{L_w}\right)$ which is stated as $(\frac{c}{L_w} - 0.07)$ in the 2006 code. Since the neutral axis depth (c) needs to be relatively larger in the 1982 design compared to the 2006 design to meet the confinement eligibility, Equation 1 gives a considerably higher value for transverse reinforcement quantity. Figure 3 displays the change of effective area of transverse reinforcement (Ash) in the boundary region versus $\frac{c}{L_w}$ for the typical walls designed according to the 1982 and 2006 standards which shows a considerably higher value of Ash for the 1982 standard. The threshold for eligibility of the confinement provisions (cc) is also indicated for both standards, which shows that the calculated neutral axis position is less than c_c for the 1982 standard, and the eligibility for confining the boundary region with total transverse reinforcement area of A_{sh} is not met; otherwise, a greater amount of transverse reinforcement for the confined boundary region would have been required for the 1982 standard when compared to the 2006 standard.

It is noteworthy that the ratio between gross area and core area ($\frac{A_g^*}{A_c^*}$) of the confinement region is controlling the calculated A_{sh} with change of the wall thickness. As the minimum cover is constant, the decrease of wall thickness results in a greater ratio which increases the calculated A_{sh}.

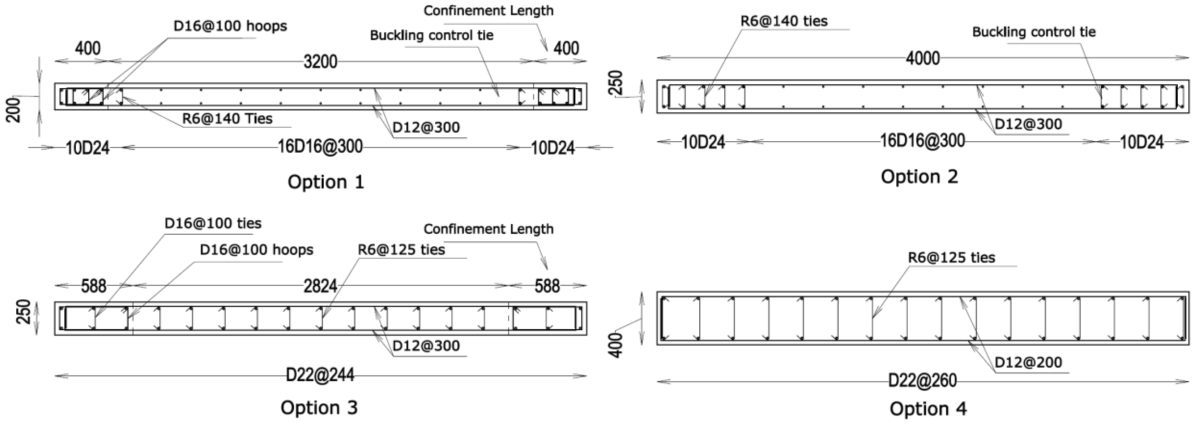


Figure 2. Design options of the typical wall according to NZS3101:1982

$$A_{sh} = \max\left\{\begin{matrix} 0.3 s_h h''\left(\frac{A_g^*}{A_c^*}-1\right)\frac{f_c'}{f_{yh}}\left(0.5+0.9\frac{c}{L_w}\right) \\ 0.12 s_h h''\frac{f_c'}{f_{yh}}\left(0.5+0.9\frac{c}{L_w}\right)\end{matrix}\right\} \quad (1)$$

$$A_{sh} = \alpha s_h h''\frac{A_g^*}{A_c^*}\frac{f_c'}{f_{yh}}\left(\frac{c}{L_w}-0.07\right) \quad (2)$$

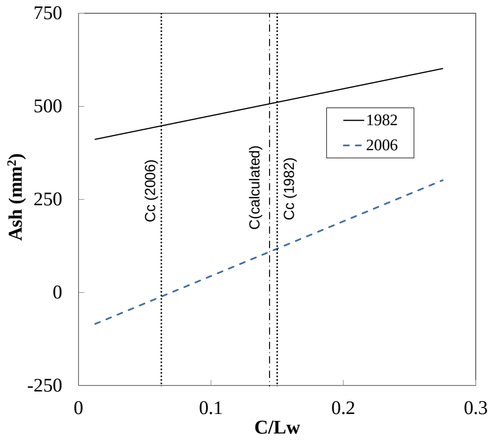


Figure 3. Total effective area of transverse reinforcement (A_{sh})

Figure 4 displays wall detailing of the Crown Plaza building (year built: 1980-1989), a ten storey commercial building damaged in the 2011 Christchurch earthquake. Depending on whether the reinforcement is uniformly distributed or concentrated more in the boundary region and also the wall thickness, the confinement region provisions of the 1982 version of the standard may result in no confinement requirement in the boundary region. Since the neutral axis position (c) controls the confinement transverse reinforcement and needs to be deep enough to meet the requirement of confinement reinforcement, the required confinement transverse reinforcement becomes significantly high especially when a relatively small thickness is adopted for the wall. The confinement provisions of this version of the concrete structures standard seem to be appropriate for the uniform distribution of longitudinal reinforcement and relatively thick walls which has enough space to accommodate the required transverse reinforcement.

Comparative analysis

Nonlinear responses of the walls designed in accordance with different versions of the New Zealand concrete design standard are compared in this section. First, section analysis is carried out to generate the moment-curvature response for each section. Then, the walls are modeled and analyzed in a FEM program to obtain their push-over as well as cyclic curves.

Section analysis

The moment-curvature curves of the wall sections generated using Xtract (TRC 2011) are shown in Figure 5a which clearly displays the substantial deficiency of the 1970 wall in terms of curvature ductility. As expected, the 2006DPR and 1995 sections with better confinement (the larger amount of hoops and smaller spacing) sustained larger curvature before failure. The curvature ductility of the 2006LDPR section is very close to the one of 2006DPR and 1995 although is designed for a lower ductility. The reason would be the smaller thickness of the 2006LDPR design which has resulted in a transverse reinforcement ratio close to the ones of 2006DPR and 1995 designs.

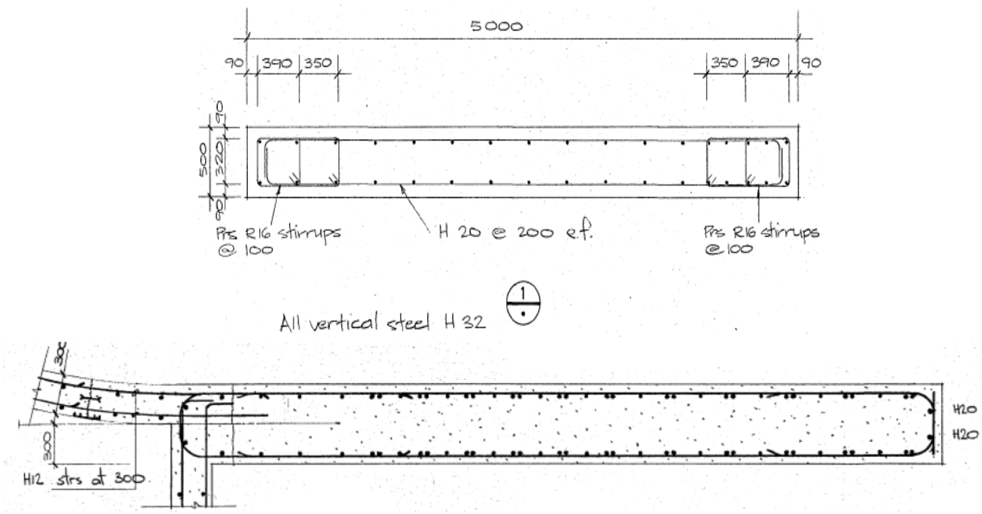


Figure 4. Wall detailing of the Crown Plaza building (1980-1989)

FEM analysis

The walls are modeled in DIANA9.4.4 (DIANA 2011). Curved shell elements with embedded bar elements are used to simulate the reinforced concrete section. The modelling approach has been comprehensively described and verified using test results of specimens with different failure modes by Dashti et al. (2014a).

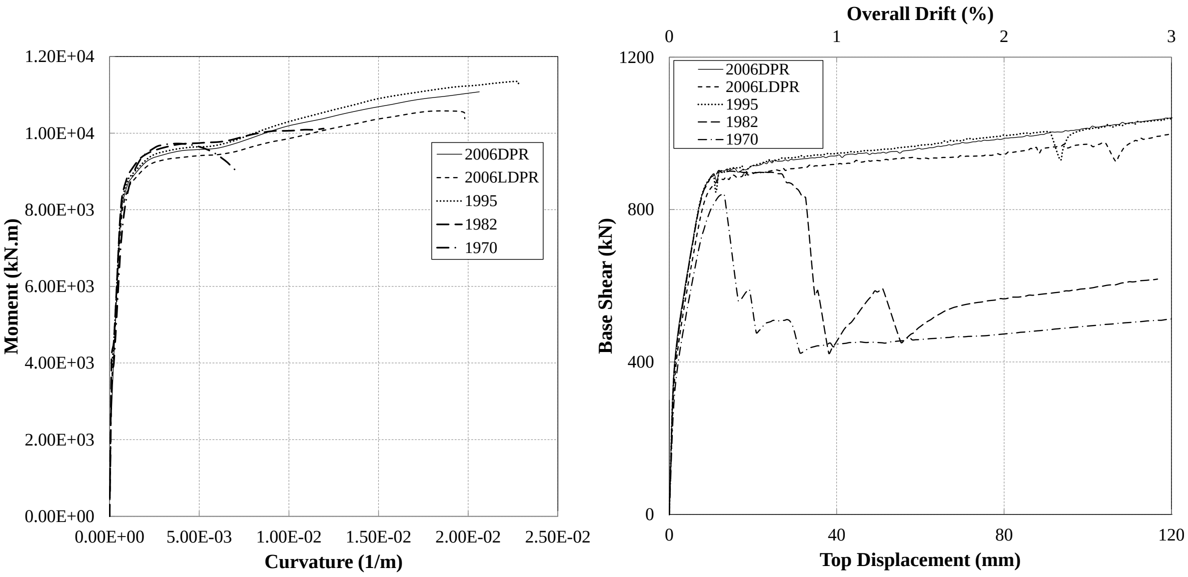
The modelling approach has been used for monotonic and cyclic response evaluation of the wall models. The mesh size is chosen based on a mesh sensitivity analysis as well as the ratio between element size and wall length of the verified specimens. Out-of plane support is provided at the story levels and a simplified displacement-controlled analysis is carried out with an incremental displacement applied at the top of the wall.

Figure 5b displays the base shear versus top displacement response of the walls. As shown in this figure, the 1970 wall undergoes a brittle failure at about 0.3% average drift. Failure of the 1982 model is also accompanied by sudden degradation of the push-over curve, but the failure displacement is greater than twice of the 1970 model. The models designed based on the 2006DPR, 2006LDPR and 1995 concrete codes are ductile enough not to fail within the range of the analysed displacement (i.e. 3.0% average drift). It should be mentioned that reinforcement buckling and bond-slip failure are not considered in these models, although geometric nonlinearity was activated in the analysis to take the P-delta effect into account.

Figure 6 displays the cyclic response of the walls. As can be seen in this figure, cyclic response of the walls designed in accordance with the 2006DPR, 2006LDPR and 1995 standards does not show any kind of degradation while the one of the 1982 and 1970 standards exhibit significant strength degradations which are due to the combined effect of flexural and shear failures.

It should be noted that as the capacity design concept was not incorporated in the 1970 code, the shear demand does not account for the flexural overstrength. Therefore, the calculated shear reinforcement was less than the minimum required by the code, and shear reinforcement configuration of the 1970 design was based on the minimum requirement.

The pushover and cyclic responses correspond well with the moment-curvature response of the wall models. However, the gap among displacement ductilities captured by member analysis of the walls is considerably higher than the one among curvature ductilities captured by section analysis which is obviously due to the effect of shear actions that can be represented in member analysis only. Although the wall designed according to the 2006DPR, 2006LDPR and 1995 codes performed well in terms of drift capacity, their response was accompanied by slight out-of-plane displacements which could have been a major issue if the walls had been subjected to more cycles at each drift level.



(a) (b)

Figure 5 (a) Moment-curvature response of the wall models (Xtract); (b) Push-over response of the wall models (DIANA)

Conclusion

The history of concrete wall design provisions in New Zealand concrete structures standards is investigated in this study by summarizing the history of each provision and designing a typical wall in accordance with these standards.

The designed walls are not necessarily representative of the corresponding version of the concrete code as they address only a limited set of design parameters. A more comprehensive parametric study needs to be conducted to derive the common configuration of concrete walls corresponding to every version of the concrete design standard.

The provisions regarding wall thickness and confinement reinforcement have been scrutinized in the designed walls. Performance of the walls is evaluated using section (moment-curvature) and member (push-over and cyclic) analyses.

İn terms of section response, the confinement in the boundary regions of walls provided fairly high curvature ductility (i.e. 8-10). Comparison of the member responses showed the effect of confinement on the displacement ductility of walls, as well.

The discrepancy among the wall models in terms of displacement ductility proved to be much greater than in the curvature ductility. This can be attributed to the effect of shear actions that cannot be captured in section analysis, by definition focusing on flexural behavior.

Although the wall designed according to the 2006DPR, 2006LDPR and 1995 versions of the concrete structures standard performed well in terms of drift capacity, their response was accompanied by slight out-of-plane displacements which, based on findings of the authors on the parameters triggering out-of-plane instability of walls (Dashti et al. 2014b), could have been a major issue if the walls had been subjected to more cycles at each drift level.

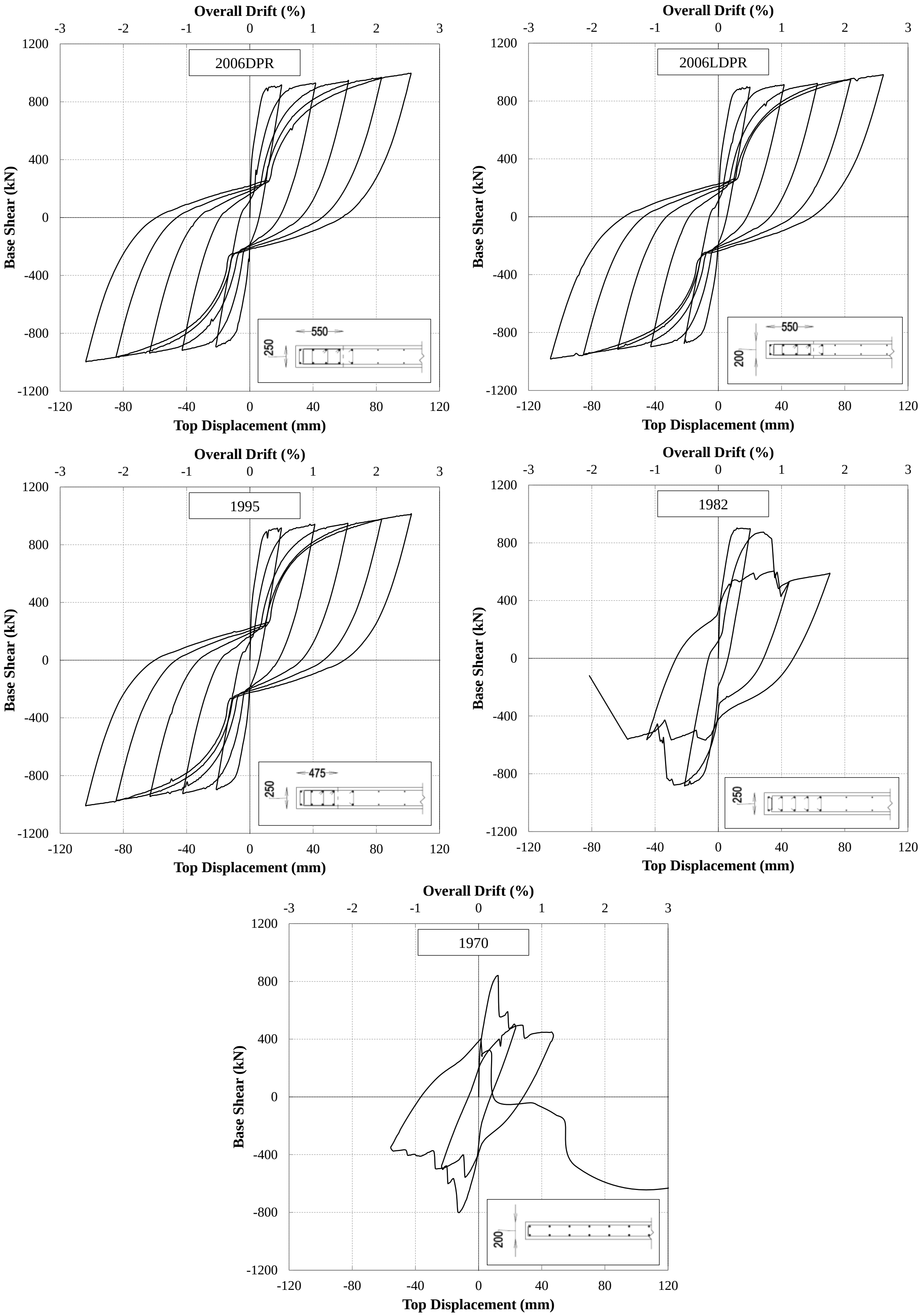


Figure 6 Cyclic response of the wall models

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Appendix

Table 4. History of New Zealand wall design provisions

Requirement	NZS 3101:2006	NZS 3101:1995	NZS 3101:1982	NZS 3101P:1970*	NZS1900: 1964 (bylaw)
Minimum thickness-general	100 mm	100 mm for the uppermost 4m of wall height and for each successive 7.5m downward(or fraction thereof), shall be increased by 25 mm.	150 mm for the uppermost 4m of wall height and for each successive 7.5m downward(or fraction thereof), shall be increased by 25 mm.	6 in.	5 in.
Limitations on the height to thickness ratio	If $N^* > 0.2f_c'A_g$ $\frac{K_e L_n}{t} \leq 30$ L_n: the clear vertical distance between floors or other effective horizontal lines of lateral support	If $N^* > 0.2f_c'A_g$ $\frac{L_n}{t} \leq 25$	$\frac{L_n}{t} \leq 10$ UNLESS: 1- the neutral axis depth for the design loading ≤4b or 0.3lw, 2- Any part of the wall within a distance of 3b from the inside of a continuous line of lateral support provided by a flange or cross wall.	$\frac{L_n}{t} \leq 35$ L_n: the distance between lateral supports (Horizontal or Vertical)	$\frac{L_n}{t} \leq 24$ L_n: the distance between lateral supports (Horizontal or Vertical)
Singly reinforced walls **Limitations on the height to thickness ratio to prevent flexural torsional buckling of in-plane loaded walls**	$\frac{k_{ft}L_n}{t} \leq 12\sqrt{\frac{L_n/L_w}{\lambda}}$ Where $N^* \leq 0.015f_c'A_g$ and $\frac{L_n}{t} \leq 75$ and $\frac{k_{ft}L_n}{t} \leq 65$	No limitations	No limitations	No limitations	No limitations
Doubly reinforced walls **Moment magnification required when:**	$\frac{k_e L_n}{t} \geq \frac{\alpha_m}{\sqrt{\frac{N^*}{f_c'A_g}}}$	No requirements	No requirements	No requirements	No requirements
Minimum thickness for prevention of instability within plastic hinge region	$b_m = \frac{\alpha_r k_m \beta(A_r + 2)L_w}{1700\sqrt{\xi}}$ $\beta = 7\ (DPR)$ $\beta = 5\ (LDPR)$	$b_m = \frac{k_m(\mu + 2)(A_r + 2)L_w}{1700\sqrt{\xi}}$	No requirements	No requirements	No requirements
Ductile detailing length -special shear stress limitations	$\max\{L_w, 0.17\frac{M}{V}\}$ Measured from the 1st flexural yielding section Need not be greater than 2Lw	$\max\{L_w, \frac{h_w}{6}\}$ Measured from the 1st flexural yielding section Need not be greater than 2Lw	$\max\{L_w, \frac{h_w}{6}\}$ Measured from the 1st flexural yielding section Need not be greater than 2Lw	No requirements	No requirements
Limitation on the use of singly reinforced walls	$\rho_l \leq 0.01$ $b \leq 200$ mm	$b \leq 200$ mm $\mu \leq 4$	$b \leq 200$ mm or if the design shear stress $\leq 0.3\sqrt{f_c'}$	Earth retaining walls: b<10 in. Other walls: b<9in	t < 10 in
Minimum longitudinal reinforcement ratio	$\rho_n = \frac{\sqrt{f_c'}}{4f_y}$	$\rho_l = \frac{0.7}{f_y}$	$\rho_l = \frac{0.7}{f_y}$	$\frac{9000}{f_y}\% \geq 0.18\%$ Note: f_y in units of [psi]	0.0025(mild steel) 0.0018 (high tensile steel)
Maximum longitudinal reinforcement ratio (ρ_l)	$\frac{16}{f_y}$	$\frac{16}{f_y}$	$\frac{16}{f_y}$	No requirements	No requirements
Maximum spacing of longitudinal reinforcement	Min{Lw/3, 3t, or 450mm}	Min {2.5b, 450mm}	Min{2.5b, 450mm}	Min{2.5b, 18in (457mm)}	2.5b

Requirement	NZS 3101:2006	NZS 3101:1995	NZS 3101:1982	NZS 3101P:1970*	NZS1900: 1964 (bylaw)
Anti-buckling reinforcement (Outside of the potential plastic hinge region)	Where $\rho_l > \begin{Bmatrix} \frac{2}{f_y} \text{ DPR} \\ \frac{3}{f_y} \text{ LDPR} \end{Bmatrix}$ dtie > db/4 Spacing $< 12d_b$	Where $\rho_l > \frac{2}{f_y}$ dtie > db/4 Spacing $< 12d_b$	Hoop or tie sets Spacing $\leq \min \begin{Bmatrix} \text{least lateral dimension} \\ 16d_b \\ 48d_{\text{transverse bar}} \end{Bmatrix}$	No requirements	No requirements
Anti-buckling reinforcement (Within the potential plastic hinge region)	Where $\rho_l > \begin{Bmatrix} \frac{2}{f_y} \text{ DPR} \\ \frac{3}{f_y} \text{ LDPR} \end{Bmatrix}$ $A_{te} = \frac{\sum A_b f_y}{96 f_{yt}} \frac{s}{d_b}$ Spacing $\leq \begin{Bmatrix} 6d_b \text{ (DPR)} \\ 10d_b \text{ (LDPR)} \end{Bmatrix}$	Where $\rho_l > \frac{2}{f_y}$ $A_{te} = \frac{\sum A_b f_y}{96 f_{yt}} \frac{s}{d_b}$ Spacing $\leq 6d_b$	Where $\rho_l > \frac{2}{f_y}$ $A_{te} = \frac{\sum A_b f_y}{96 f_{yt}} \frac{s}{100}$ Spacing $\leq 6d_b$	No requirements	No requirements
Confinement reinforcement	Where neutral axis depth > $c_c = \frac{0.1\phi_{ow} L_w}{\lambda}$ $\lambda = 1.0(\text{DPR})$ $\lambda = 2.0(\text{LDPR})$ $A_{sh} = \alpha s_h h'' \frac{A_g^*}{A_c^*} \frac{f_c'}{f_{yh}} \left(\frac{c}{L_w} - 0.07\right)$ $\alpha = 0.25(\text{DPR})$ $\alpha = 0.175(\text{LDPR})$	Where neutral axis depth > $c_c = \left(\frac{0.3\phi_o}{\mu}\right) L_w$ $A_{sh} = \left(\frac{\mu}{40} + 0.1\right) s_h h'' \frac{A_g^*}{A_c^*} \frac{f_c'}{f_{yh}} \left(\frac{c}{L_w} - 0.07\right)$	Where neutral axis depth > $c_c = \begin{Bmatrix} 0.1\phi_o S l_w \\ \text{or} \\ \frac{8.6\phi_o S l_w}{(4-0.7S)\left(17+\frac{h_w}{l_w}\right)} \end{Bmatrix}$ $A_{sh} = \max \begin{Bmatrix} 0.3 s_h h'' \left(\frac{A_g^*}{A_c^*} - 1\right) \frac{f_c'}{f_{yh}} \left(0.5 + 0.9\frac{c}{L_w}\right) \\ 0.12 s_h h'' \frac{f_c'}{f_{yh}} \left(0.5 + 0.9\frac{c}{L_w}\right) \end{Bmatrix}$	No requirements	No requirements
Maximum spacing of confinement reinforcement	DPR: min{6db, 0.5t} LDPR: min{10db, t}	Min {$6d_b$, 0.5t, 150mm}	Min {$6d_b$, 0.5t, 150mm}	No requirements	No requirements
Minimum confinement length	$\max \begin{Bmatrix} c - 0.7c_c \\ 0.5c \end{Bmatrix}$ c: neutral axis depth	$\max \begin{Bmatrix} c - 0.7c_c \\ 0.5c \end{Bmatrix}$ c: neutral axis depth	0.5c	No requirements	No requirements
Maximum nominal shear stress	$v_n \leq 0.2f'_c$ or 8MPa	$v_n \leq \begin{cases} 0.2f'_c \\ 1.1\sqrt{f'_c} \\ 9\text{MPa} \end{cases}$	$v_n \leq 0.2f'_c$ or 6MPa	$v_u \leq (0.8 + 4.6\frac{H}{D})\phi\sqrt{f'_c}$ $v_u \leq 5.4\phi\sqrt{f'_c}$ for H/D<1 $v_u \leq 10\phi\sqrt{f'_c}$ for H/D>2 $\phi = 0.85$	$v = \frac{f_c}{1 + \frac{h^2}{49t^2}}$
Concrete shear strength(Simplified)	$V_c = \min \begin{Bmatrix} 0.17\sqrt{f'_c} A_{cv} \\ 0.17\left[\sqrt{f'_c} + \frac{N^*}{A_g}\right] A_{cv} \end{Bmatrix}$	$v_c = \min \begin{Bmatrix} 0.2\sqrt{f'_c} \\ 0.2\left[\sqrt{f'_c} + \frac{N^*}{A_g}\right] \end{Bmatrix}$	$v_c = \min \begin{Bmatrix} 0.2\sqrt{f'_c} \\ 0.2\left[\sqrt{f'_c} + \frac{P_u}{A_g}\right] \end{Bmatrix}$	The shear stress carried by the concrete shall not exceed: $v_c = \left(3.7 - \frac{H}{D}\right) 2\phi\sqrt{f_c'}$ $v_c \leq 5.4\phi\sqrt{f'_c}$ for H/D<1 $v_c \leq 2\phi\sqrt{f'_c}$ for H/D>2.7 $\phi = 0.85$	No requirements
Shear reinforcement	$A_v = V_s \frac{s_2}{f_{yt} d}$	$A_v = \frac{(v_n - v_c) b_w s_2}{f_{yt}}$	$A_v = \frac{(v_n - v_c) b_w s_2}{f_{yh}}$	$A_v = \frac{V_u' s}{\phi f_y d(\frac{H}{D} - 1)}$	No requirements
Minimum shear reinforcement	$A_v = \frac{0.7 b_w s_2}{f_{yt}}$	$A_v = \frac{0.7 b_w s_2}{f_{yt}}$	$A_v = \frac{0.7 b_w s_2}{f_{yh}}$	$A_v = \frac{V_u' s}{\phi f_y d}$ or Ratio (%): $\frac{9000}{f_y} \geq 0.18$	0.0025(mild steel) 0.0018 (high tensile steel)

Requirement	NZS 3101:2006	NZS 3101:1995	NZS 3101:1982	NZS 3101P:1970*	NZS1900: 1964 (bylaw)
Maximum spacing of shear reinforcement	$\mathrm{Min}(\frac{L_w}{5}, 3t, \text{or } 450mm)$	$\mathrm{Min}(\frac{L_w}{5}, 3t, \text{or } 450mm)$	$\mathrm{Min}(\frac{L_w}{5}, 3t, \text{or } 450mm)$	2.5t, 18in (457mm)	2.5t
Vertical reinforcement	$\rho_n \ge \frac{0.7}{f_{yn}}$ Spacing $\le \min\{\frac{L_w}{3}, 3t, 450mm\}$	$\rho_n \ge \frac{0.7}{f_{yn}}$ Spacing $\le \min\{\frac{L_w}{3}, 3t, 450mm\}$	$\rho_n \ge \frac{0.7}{f_{yn}}$ Spacing $\le \min\{\frac{L_w}{3}, 3t, 450mm\}$	No requirements	No requirements
Maximum shear strength provided by the concrete in Ductile detailing length	$V_c = \left(0.27\lambda\sqrt{f'_c} + \frac{N^*}{4A_g}\right) b_w d \ge 0.0$ $\lambda = 0.25DPR$ $\lambda = 0.5LDPR$	v_c shall not be taken larger than: $v_c = 0.6\sqrt{\frac{N^*}{A_g}}$ Total nominal shear stress shall not exceed: $v_n = (\frac{\phi_{ow}}{\mu} + 0.15)\sqrt{f'_c}$	v_c shall not be taken larger than: $v_c = 0.6\sqrt{\frac{P_e}{A_g}}$ Total nominal shear stress shall not exceed: $v_n = (0.3\phi_o S + 0.16)\sqrt{f'_c}$ S: structural type factor as defined by NZS 4203	No requirements	No requirements
Splicing of flexural tension reinforcement	One-third (DPR) and one-half (LDPR) of reinforcement can be spliced where yielding can occur	One-third of reinforcement can be spliced where yielding can occur	One-third of reinforcement can be spliced where yielding can occur	One-half of reinforcement can be spliced where yielding can occur	No requirements
Maximum compressive stress in concrete	No requirements	No requirements	No requirements	$\left[1 - \left(\frac{h}{35d}\right)^3\right] 0.2f'_c$ h: distance between supports d: thickness of wall	Direct loading: kf_{cu} $k = \frac{p}{5} - 0.007\frac{h}{t} + 0.2$ f_{cu}: minimum crushi P: total percentage of vertical reinforcement $0.25 \le p \le 0.5$ $\frac{h}{t} \ge 10$ Seismic bending + direct stress: 1.25k
Maximum stress in the tensile steel	No requirements	No requirements	No requirements	No requirements	15000 psi for mild steel 20000 psi for the special types of reinforcement covered by the First Schedule hereto

*Note: NZS3101P:1970 units of [psi],

Notation	NZS 3101:2006	NZS 3101:1995	NZS 3101:1982	NZS 3101P:1970*	NZS1900: 1964 (bylaw)
Design axial load at the ultimate limit state	N^*	N^*	P_u	*NA*	*NA*
The clear vertical distance between floors or other effective horizontal lines of lateral support, or clear span	L_n	L_n	L_n	h	h
Wall thickness	t , b	b	b	d , b	t
Effective length factor for Euler buckling	k_e	*NA*	*NA*	*NA*	*NA*
Effective length factor for flexural torsional buckling	k_{ft}	*NA*	*NA*	*NA*	*NA*
Horizontal length of wall	L_w	L_w	l_w	D	*NA*
Thickness of boundary region of wall at potential plastic hinge region	b_m	b_m	*NA*	*NA*	*NA*
Total height of wall from base to top	h_w	h_w	h_w	H	*NA*
Aspect ratio of wall (h_w/L_w)	A_r	A_r	*NA*	*NA*	*NA*
Yield strength of non-prestressed reinforcement	f_y	f_y	f_y	f_y	*NA*
Yield strength of transverse reinforcement	f_{yh}	f_{yh}	f_{yh}	*NA*	*NA*
Yield strength of shear reinforcement	f_{yt}	f_{yt}	f_{yh}	f_y	*NA*
Ratio of vertical (longitudinal) wall reinforcement area to gross concrete area of horizontal section	$\rho_n = \frac{A_t}{A_g}$	*NA*	*NA*	*NA*	*NA*
The ratio of vertical wall reinforcement area to unit area of horizontal gross concrete section	$\rho_l = \frac{A_s}{ts_v}$	$\rho_l = \frac{A_s}{bs_v}$	$\rho_l = \frac{A_s}{bs_v}$	*NA*	*NA*
Diameter of longitudinal bar	d_b	d_b	d_b	*NA*	*NA*
Center-to-center spacing of shear reinforcement along member	s	s	s	*NA*	*NA*
Computed distance of neutral axis from the compression edge of the wall section	c	c	c	*NA*	*NA*
A limiting depth for calculation of the special transverse reinforcement	c_c	c_c	c_c	*NA*	*NA*
Overstrength factor	ϕ_{ow}	ϕ_o	ϕ_o	*NA*	*NA*
Area of concrete core	A_c^*	A_c^*	A_c^*	*NA*	*NA*
Gross area of concrete section	A_g^*	A_g^*	A_g^*	*NA*	*NA*
Dimension of concrete core of rectangular section measured perpendicular to the direction of the hoop bars to outside of peripheral hoop	h''	h''	h''	*NA*	*NA*
Center-to-center spacing of hoop sets	s_h	s_h	s_h	*NA*	*NA*
Structural type factor	----	----	S	*NA*	*NA*
Displacement ductility capacity relied on in the design	*NA*	μ	*NA*	*NA*	*NA*
Area used to calculate shear area	A_{cv}	*NA*	*NA*	*NA*	*NA*
Total nominal shear strength	V_n	V_n	V_n	*NA*	*NA*
Design shear force	V^*	V^*	V_u	V_u	*NA*
Concrete shear strength	V_c	*NA*	*NA*	*NA*	*NA*
Nominal shear strength provided by shear reinforcement	V_s	*NA*	*NA*	*NA*	*NA*
Shear stress provided by concrete	v_c	v_c	v_c	v_c	*NA*
Centre-to-centre spacing of horizontal shear reinforcement	s_2	s_2	s_2	s	*NA*